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THREE SCHEMES HAVE BEEN PROPOSED TO SATISFY THE CEASE AND DESIST ORDER OF THE STATE OF COLORADO AGAINST RMA. EACH SCHEME PROPOSES TO INTERCEPT THE NORTHERLY FLOW OF GROUND WATER, PROCESS THIS WATER THROUGH A TREATMENT SYSTEM, AND RETURN THE CLEAN WATER TO THE AQUIFER. THIS REPORT EXAMINES THE THREE PROPOSED SYSTEMS AND RECOMMENDS THE MOST TECHNICALLY FEASIBLE ROUTE FOR OBTAINING THE FINAL OBJECTIVE. IT ALSO RECOMMENDS WHETHER OR NOT PILOT PHASES SHOULD BE PURSUED.

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RESEARCH REPORT

Battelle

FINAL REPORT

on

STUDY OF ALTERNATIVES FOR GROUND WATER POLLUTION CONTROL AT THE NORTH BOUNDARY OF ROCKY MOUNTAIN ARSENAL

to

THE DEPARTMENT OF THE ARMY
PROJECT MANAGER FOR CHEMICAL DEMILITARIZATION AND INSTALLATION RESTORATION
Aberdeen Proving Ground, Maryland

January 13, 1977

by

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and

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Moody and Associates, Inc.

SUMMARY AND RECOMMENDATIONS

Three schemes have been proposed to satisfy the Cease and Desist Order of the State of Colorado against Rocky Mountain Arsenal. Each scheme proposes to intercept the northerly flow of groundwater, process this water through a treatment system, and return the clean water to the aquifer. Battelle, and Moody and Associates, were contracted by the Project Manager's Office, Chemical Demilitarization and Installation Restoration, Aberdeen Proving Ground, Maryland to (1) examine the three proposed systems, (2) to recommend the most technically feasible route for obtaining the final objective, and (3) to recommend whether or not pilot phases should be pursued.

The three systems were:

(1) A system which uses dewatering wells on a line near the north border of Rocky Mountain Arsenal to intercept the groundwater; a bentonite dam further downgradient from this line of dewatering wells to assure complete interception of all contaminated flow, and recharge pits to return the water to the aquifer downgradient from the dam.

(2) A French drain system composed of a barrier in the groundwater flow system to stop the flow of contaminated water. A horizontal drain gravel filled around a perforated pipe collects groundwater flowing into the barrier, and a similarly constructed recharge trench to return the groundwater to the aquifer. The original proposal was to place this system approximately 2500 feet south of the north border, but recent decisions have moved it to the vicinity of the north border.

(3) A hydraulic gradient system which employs a line of dewatering wells to establish a valley in the groundwater table and further downgradient, a recharge system which provides a corresponding rise in the groundwater table. The resulting hydraulic gradient between the valley and the mound produces an upgradient flow of water and traps the contaminated water between the dewatering and recharge systems. This system was originally proposed for placement 2500 feet south of the northern border, but more recent proposals place it 500 feet south.

Battelle and Moody examined the availability of data, the cost estimates, and the technical feasibility of the proposed systems, and combined the various constraints into a rating system. Based upon the above analyses and the results of the weighting system, it was determined that, on comparing the French drain and the bentonite dam systems, the bentonite dam system is superior, primarily because of its cost advantage. The hydraulic gradient was significantly lower in price than the bentonite dam. However, Battelle and Moody believe that there is significant technical risk in all of the proposed hydraulic gradient designs--there are stretches along the northern boundary which will require a bentonite dam in order to meet the 500 ppb DIMP (diisopropylmethyl phosphonate) discharge criterion.

The selection of the most appropriate system was based upon an estimated 3500-foot system needed at the north boundary to intercept essentially all of the contaminated flow. However, since only 10,000 gallons per hour of treatment capacity are currently available, the groundwater flow of roughly 1/3 of the 3500-foot alignment can be treated. This constraint, coupled with other factors, suggests that a pilot-scale system, sized to the existing treatment capacity, should be built to test out some of the unknowns that currently exist without penalty.

Battelle and Moody also examined several variations of the hydraulic gradient concept in an effort to find a more technically feasible configuration. It was determined that there are hydraulic gradient configurations which allow a higher chance of success than the originally proposed scheme. Nevertheless, the risks with these better configurations are too high to allow their recommendation for the entire alignment. Battelle and Moody recommend that the alignment be constructed in stages, with the first segment a 1500-foot bentonite dam extending westward from 1400 feet east of 'D' Street.

INTRODUCTION

Past activities of Rocky Mountain Arsenal (RMA) have resulted in some contamination of the groundwater system flowing through the Arsenal. This condition has resulted in the State of Colorado issuing a Cease and Desist Order against RMA. The Army is now working to develop a system that will intercept the contaminated portion of the groundwater aquifer, process this water through a treatment system, and return the water to the groundwater aquifer at or near the point where the groundwater flow leaves the Arsenal. Battelle-Columbus and Moody and Associates, Inc. were contracted by the Project Manager, Chemical Demilitarization and Installation Restoration (PM CDIR), to examine three proposed systems and to recommend the best system to satisfy the Cease and Desist Order. This report summarizes the findings of the study commissioned by PM CDIR.

Battelle and Moody examined the available data on the aquifer at RMA to determine whether there are gaps in or procedural problems in the generation of these data which would disallow any of the proposed systems. It is shown in this report that problems so identified will not interfere with the pilot scale installation of the recommended system.

A cost analysis was made of each of the three systems proposed. Costs show that a hydraulic gradient concept is the least expensive, in terms of capital costs, followed by the bentonite dam, and finally by the French drain. The costs ignored operating and maintenance costs (by directive from the Army), and rather stressed only the capital cost requirements.

A technical review was made of the three systems. Where there are some potential failure modes in the bentonite dam and French drain systems, the probability of failure is very low. In consequence, those systems will not allow the escape of contaminated water past barriers. The hydraulic gradient system shows a very high probability of leaking; that is, contaminated water will flow past (or in between) the dewatering wells. Furthermore, there are certain side effects of the hydraulic gradient system as proposed which would need to be considered in the final design of the system.

Four goals were identified for the system, as follows:

(a) Build and operate a system by October 1977. This operable system would not necessarily address the entire contaminated northern boundary, but could, as a pilot system, show the Army's desire and intent to address the Cease and Desist Order.

(b) Eventually, if not immediately, place a groundwater cleanup system across the entire north alignment to capture essentially the entire flow of contaminated water.

(c) Do the above with a minimum of expenditures.

(d) Avoid changing or disrupting the natural water tables to the area north of the north boundary of RMA.

These goals were used explicitly in the selection of the recommended system.

The final piece of this report has shown that there are certain oversights which ought to be addressed in an Environmental Impact Statement (EIS) for the sake of completeness.

RESULTS

During the conduct of the study a review was made of the data available, the estimated cost and the technical feasibility of the three options. The Battelle and Moody team found some problems with the water data taken by the Waterways Experiment Station, a probable feasibility problem with one of the three options, and oversights which need to be considered in the presentation of any EIS.

DATA REVIEWED

The Waterways Experiment Station (WES) established drawdown curves for the wells across the northern boundary based upon a transient

400 minute test. Based upon the results of the tests they estimate a maximum flow of 10,000 gallons per hour across the 1400-foot alignment examined.

The data collected by the WES was examined through the use of the Aron-Scott technique (or its equivalent, the Jacob technique). Neither of these techniques is applicable to the transient data that have been taken by the WES.

The reexamination of these data was undertaken using a transient analysis. Based upon the results of the transient analysis on three wells for which acceptable data exists, it was determined that the storage coefficient at the three wells was 0.05 while the permeability ranged between 120 to 230 ft per day, as compared to the WES figures of 205 ft per day. However, the figure of 10,000 gallons per hour peak flow across the 1400 ft alignment is believed to be correct. Figures 1, 2, and 3 are attached to show the correct transient analysis and the results thereof.

An analysis of local water table slopes across the two alignments examined by WES show local slopes between .005 and .015. The numbers used by WES were .005 to .010 with the average value selected by WES of .0094.* More recent data taken at Rocky Mountain suggests that the local water table slope further south of the north boundary is smaller while the thickness of the aquifer is greater. If the product of the permeability, aquifer thickness, and local water table slope are representative of groundwater flow rates, then, for the aquifer crossing the northern boundary, this product must be the same averaged across the northern boundary as it is averaged across an alignment further back from the northern boundary (ignoring precipitation gains). While no fault is found with the roughly determined values of water table slope further back from the boundary, it is believed that a critical value for an analysis of the hydraulic gradient system, is the product of aquifer thickness, permeability and water table slope, a value which we believe is fairly well determined by the results of the WES experiments.

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* The values of Konikow data for water table slope appear to be about .01. These numbers were taken in earlier years and changes in precipitation may account for differences in measured table slopes.

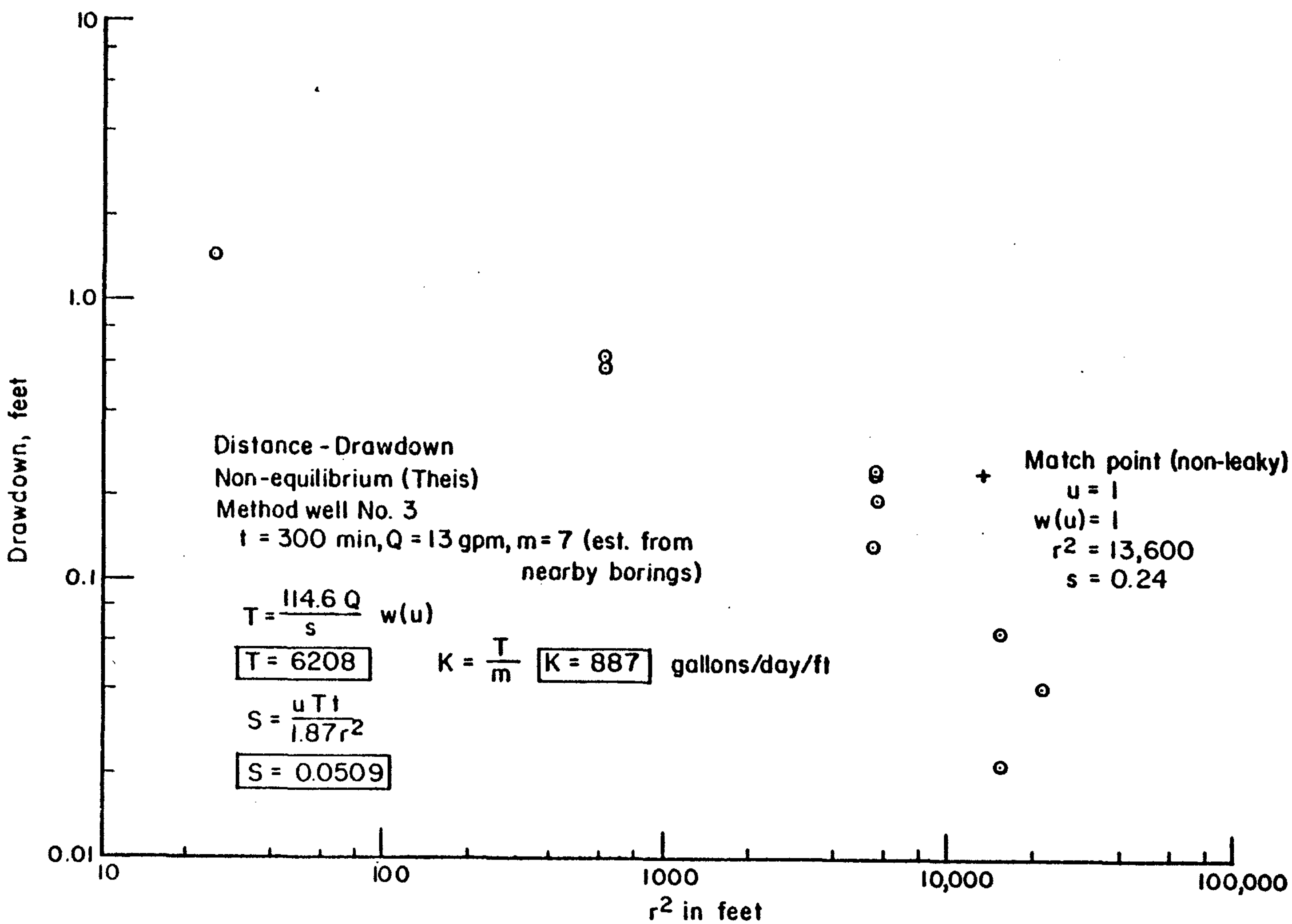


FIGURE 1. DISTANCE - DRAWDOWN CURVE, WELL 3

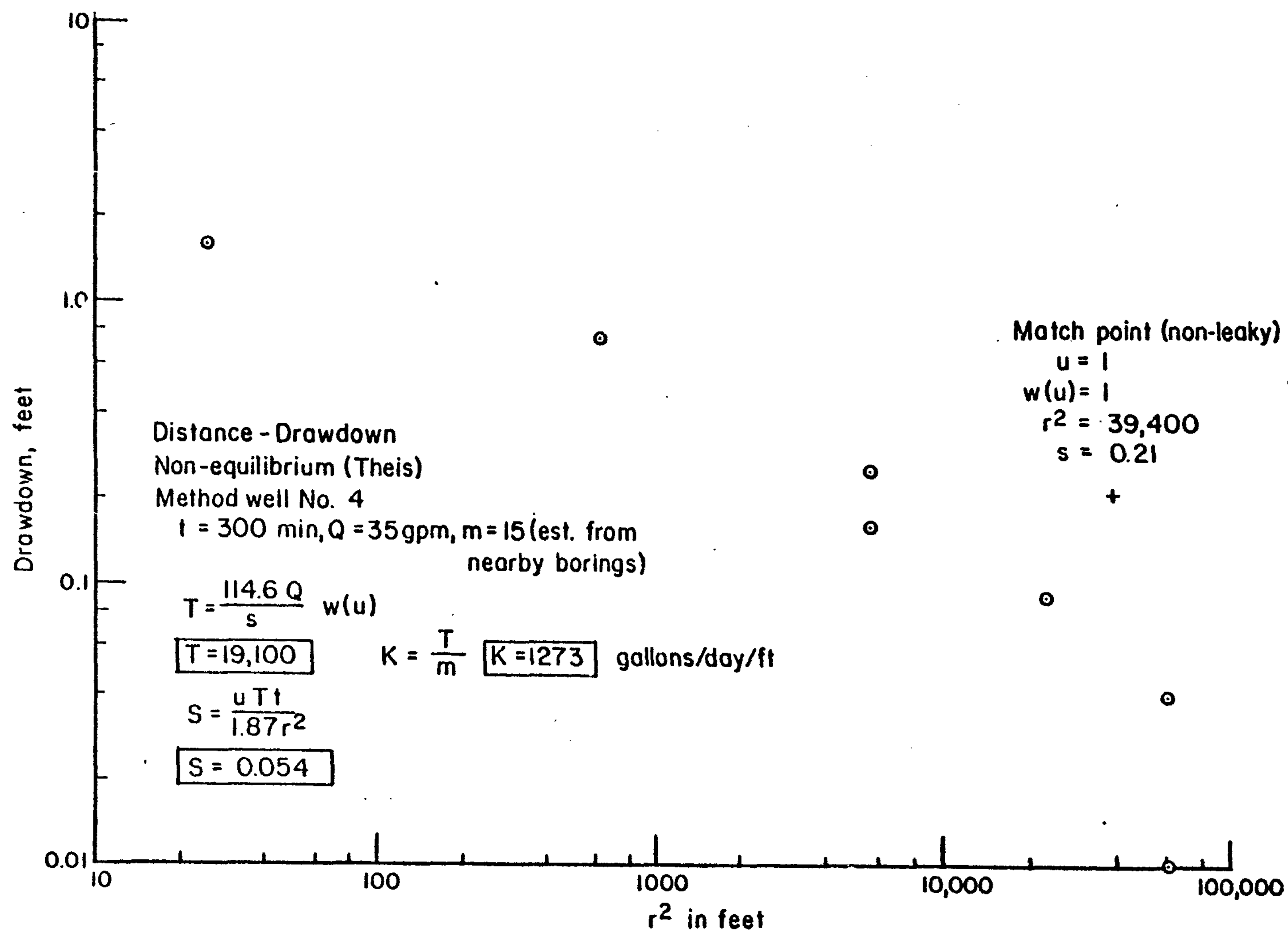


FIGURE 2. DISTANCE DRAWDOWN CURVE, WELL 4

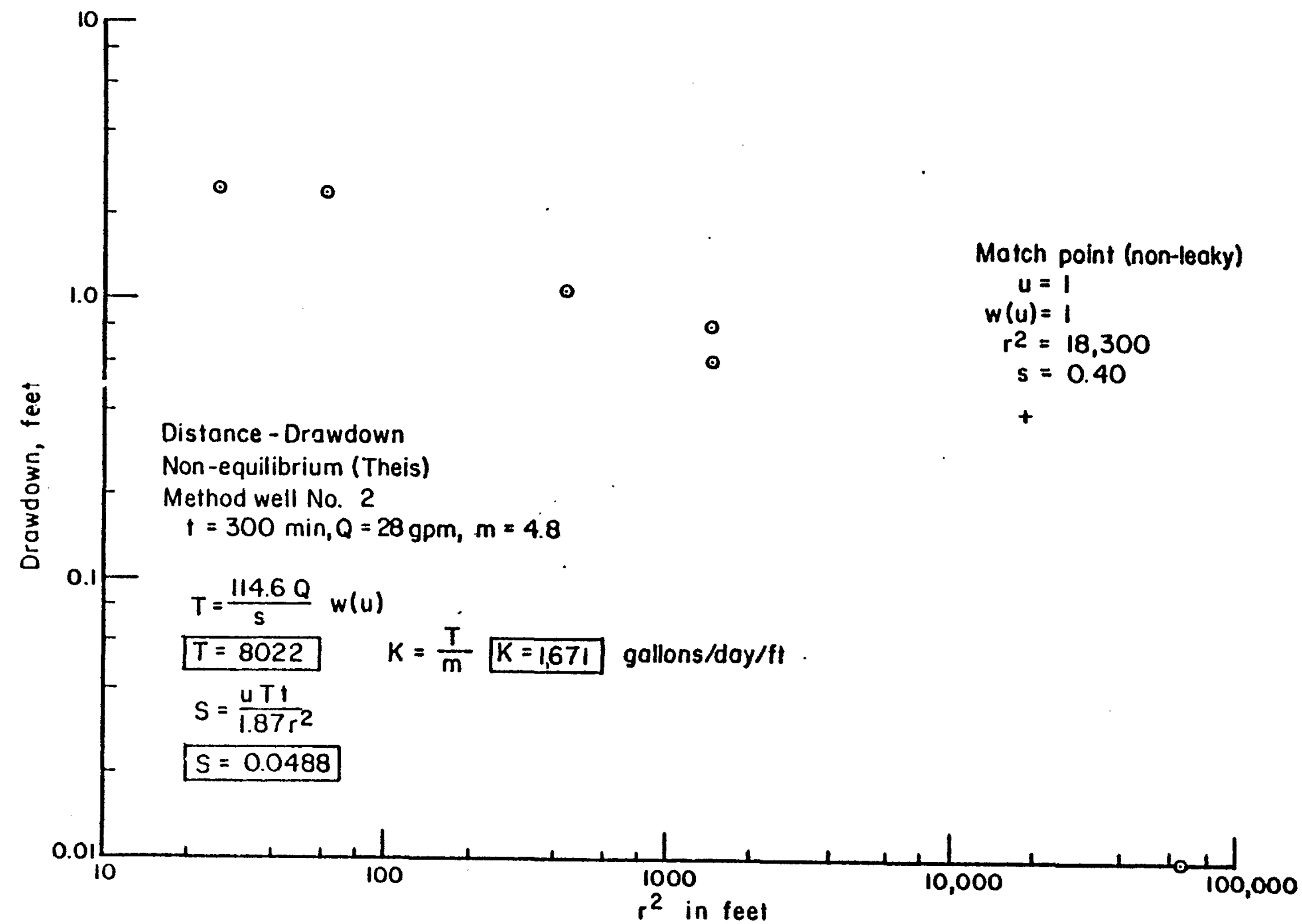


FIGURE 3. DISTANCE DRAWDOWN CURVE, WELL 2

Fault was found with the procedure by which WES determined the number of wells to be used across the 1400-foot alignment. Figures 4 and 5 show an analysis of the functionability of the WES concept for a varying number of dewatering wells. These figures show graphically that the WES concept could operate with four wells instead of seven.

One question which came up in meeting with RMA and PM CDIR personnel was that of the length of dewatering alignment necessary to completely intercept the contaminated flow across the northern boundary. We used a value of 3500 ft—a cross sectional length which captures a majority of the flow which is considered to be contaminated. By combining permeability of the saturated aquifer, permeability of the overlying sand/silt layer, and contaminant concentration, it was possible to determine the local mass rate flow of contaminant across the northern boundary per running foot of alignment per unit slope. The results of this analysis are present in Figure 6. It is seen from this figure that the proposed 3500-foot alignment is amply capable of intercepting essentially all of the actual contaminant flow for the northern boundary. Extensions of the alignment to either side will allow the capture of even more contaminant, but the volume of contaminants captured by these extensions is insignificant compared to that already being captured by the 3500-foot alignment. If extension to the west is desired, wells would not capture the water (due to low permeability) and an open trench should be considered.

Capital Cost Estimates

PM CDIR personnel directed Battelle to examine only the capital costs of the proposed systems and not to consider the capital or operating cost of the cleanup facilities which were to be provided as a black box. The cost of monitoring wells were ignored because it is believed that monitoring requirements will be similar for the three proposed systems.

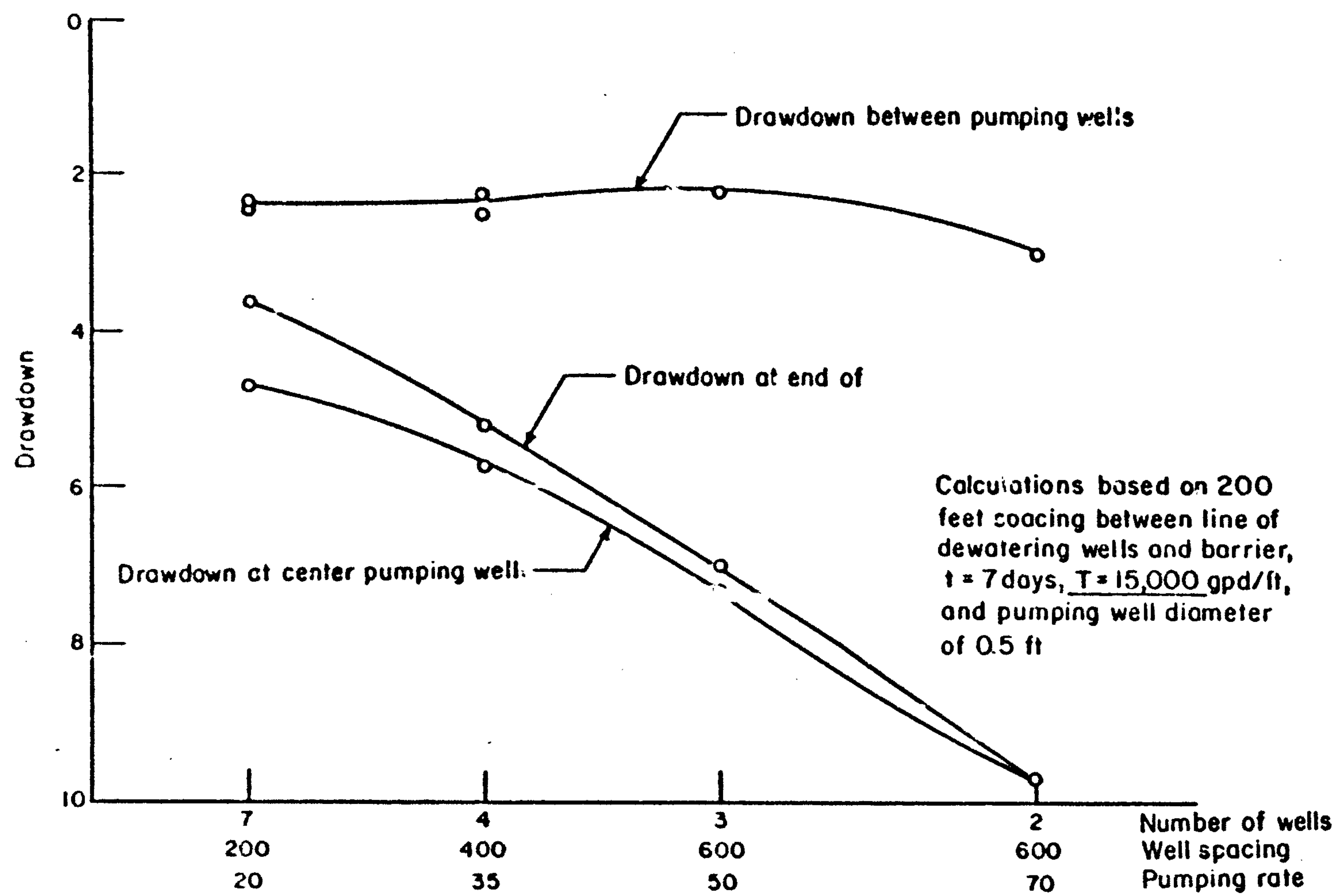


FIGURE 4. BENTONITE DAM, T = 15000

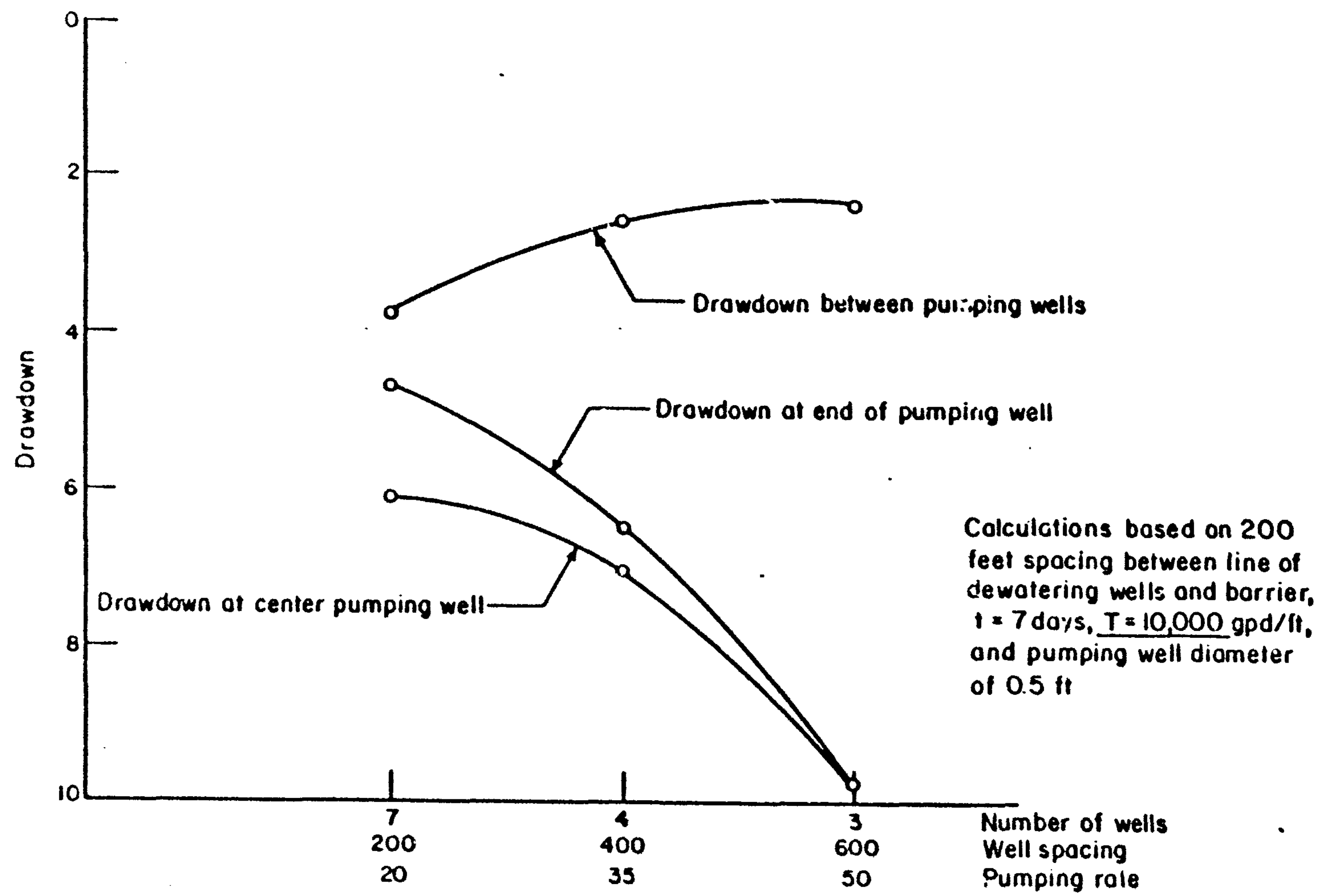


FIGURE 5. BENTONITE DAM, T = 10000

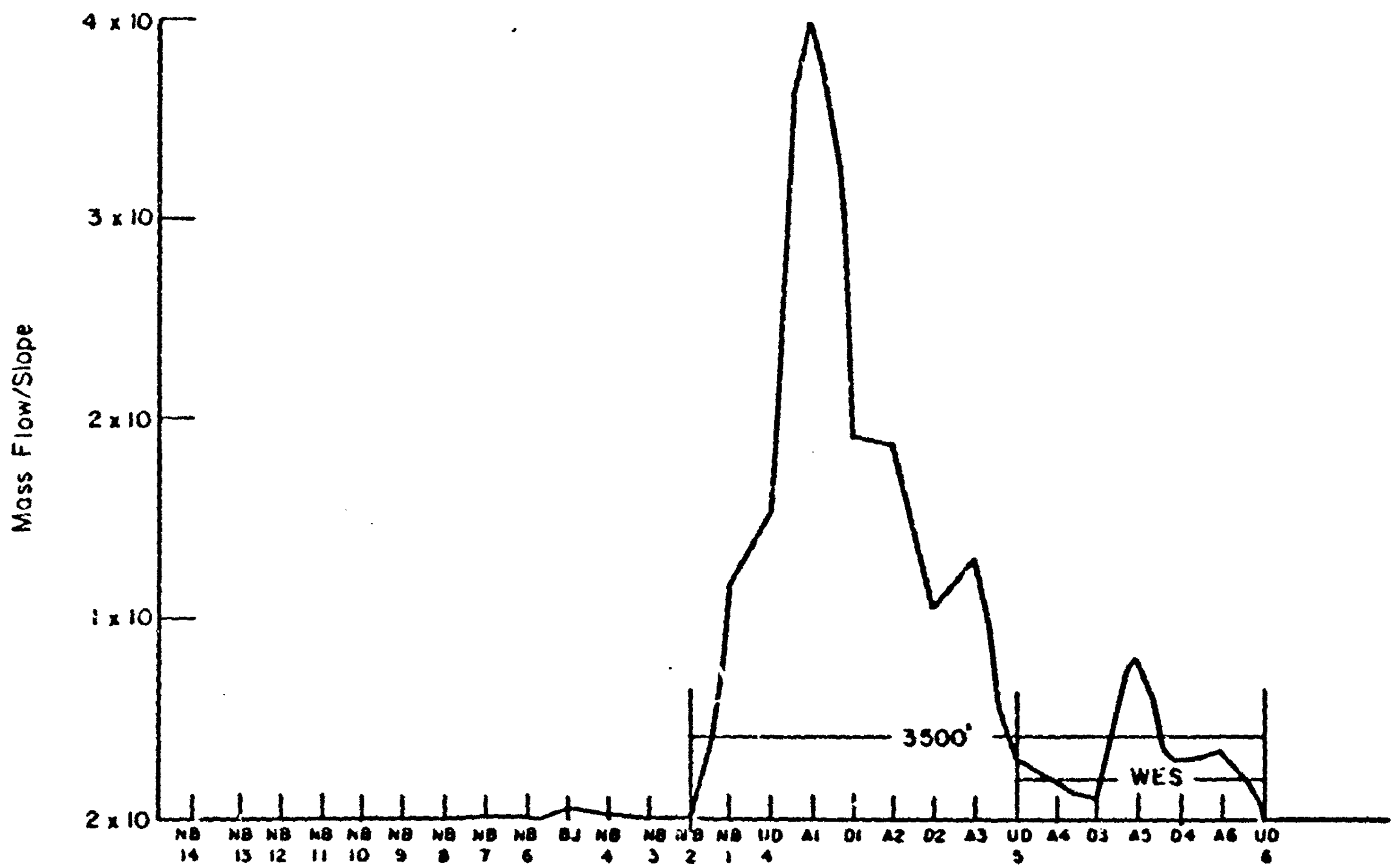


FIGURE 6. FT^3/DAY-FT/UNIT SLOPE VS WELL NUMBER

The wells for all systems were assumed to be 8-inch wells with each well having a pump and a device to maintain a head of a constant level.*

Costs for the bentonite dam system and the French drain system were based upon both a 1400-foot and 3500-foot alignment, while the costs for the hydraulic concept were based upon the original alignment proposed by Rocky Mountain Arsenal.

Costs for the three alternatives are presented in Tables 1, 2, and 3. The primary cost for the bentonite dam system is that of the bentonite dam itself. Estimates for installing this trench range from $50 to $110 per*** ft. A figure of $2.80 per square foot was chosen as being representative: actual costs may vary ± 55¢ (± 20%). Estimates for the French drain construction range from $130 to $250 per running ft. A cost of $200 a ft was chosen as being representative. This construction cost is based upon the necessity of cutting a V-shape trench at an angle of repose of 45 degrees to accomplish the construction of the trench. The use of trench boxes was examined and discarded because of:

(a) The need to use several vertical sections of trench boxes, and

(b) The problems of advancing trench boxes over significant lengths of alignment.

The costs of the hydraulic gradient concept do not require further explanation.

With the caveats and considerations discussed above, the capital cost required to intercept and treat the contaminated flow of water across the northern boundary for the three proposed systems** are is follows:

(a) The bentonite dam system should cost $427,440

(b) The French drain system should cost $866,500

(c) The hydraulic gradient system should cost (as proposed) $261,900.

---

* 8" Wells were selected to provide room for the head maintaining equipment. Gravel pack was originally considered, but has since been discarded as unnecessary.

** Engineering costs should add 10% to these figures.

*** $1.85 to $4.07 per sq ft.

TABLE 1. ALTERNATIVE #1 BENTONITE TRENCH

		1400'	3500'
(1) Slurry Trench $2.80/ft^2 x 27' deep x 1400'		105840	$264,600
(2) Wells 8" (4 wells)		25600	10 Wells- $64,000
Drilling $12/ft x 24'	$325		
Casing $12/ft x 27'	325		
Screen $65/ft x 5'	325		
Development 8 hrs x $90	720		
Pump test	400		
Chart & pad	330		
Total	2500		
Pump & access	1400		
Control equipment	2500		
Recharge pit (Bog)		30000	45000
		$161,440	$373,600
Other Costs			
Dewatering Transmission	46,800		
Elec. Trans.	12,000		
Collector pipe	14,700		
Elec. Dist.	7,500		
Dist. pipe - 0	0		
	81,000	$242,440	$477,000

TABLE 2. ALTERNATIVE #2 FRENCH DRAIN

French Drain	
French construction and backfill materials @$200/ft	$280,000
Perforated pipe (culvert pipe (18") + labor) @ $12/ft $11/ft + $1/ft	16,800
Liner on down-gradient side (hypalon, etc)	
.8 Acres x $5000/acre = $4000	4,000
Riser pipes - 20" ($20/ft x 50') = $1000	1,000
Pumps - 2 turbine (150 gallon cap) @ $2500	5,000
(5 hp pumps and accessories)	$306,800
Power	
Controls	
Pipeline	
Recharge Trench	
Trench construction and backfill materials	$25,000
Perforated pipe (as above)	16,800
	$41,800
Other Costs	$30,500
1400' total	$379,100
3500' =	947,750

This alternative will require less operating and maintenance cost.

TABLE 3. ALTERNATIVE #3 HYDRAULIC GRADIENT (2500' LINE SETBACK) 275' ON CENTER

Dewatering wells and equipment		
17 x $7600		$129,200
Recharge		45,000
Other		
Dewatering trans	$0	
Elect. trans	0	
Collector pipe	49,100	
Elect. dist.	11,700	
Recharge trans	23,400	
Dist. pipe	3,500	87,700
	$87,700	$261,900

TECHNICAL RISK

In order to select a system for installation it is necessary to consider the technical risk of applying the system. This risk not only includes the known possibilities of failure due to electrical failure and chronic deterioration of components, but also any design oversights which might cause the system to operate in a less than satisfactory mode. The technical discussion presented below considers the three alternatives in the alignment necessary to capture the entire flow of contaminated water.

The bentonite dam system in its presumed mode of operation suffers from little technical risk. In the event of failure of individual components, the dam itself should act as a storage medium for a period of several days to trap the flow of contaminated water. After repair and replacement of affected components contaminated water which has backed up behind the dam can be captured by the dewatering wells. A strong possibility in a dam of this length is that of bridging the dam because of clay and incorrectly mixed materials within the dam. In this case the hydraulic differential across the dam would serve to force clean water "upgradient" preventing the escape of contaminated water. The net result would be to increase slightly the total water processed by the treatment plant. The same analysis holds for possible seepage underneath the dam. The worst possible failure mode would be that in which the entire dewatering alignment fails for a lengthy period of time (days). In this case the water backing up behind the bentonite dam would rise slowly to surface and would probably breech over the dam, creating a very wet and sticky surface in the vicinity of the dam, and considerable leakage.

The analysis discussed above for the bentonite dam is essentially the same for the French drain system. However, in the French drain system the problem is not that of breeching, but rather of failure of the plastic barrier. This leakage could occur because of (1) chemical reaction with the plastic, which is probably likely over a long period of time, or (2) of penetration of the plastic by sharp objects in the back-filled materials. The likelihood of both of these circumstances can be significantly decreased by the use of a suitable plastic liner such as Hypalon. The more probable failure mode of the French drain system is that of lifting the lower edge of the plastic sheet and the subsequent flowing of fresh water underneath the plastic sheet upgradient to be captured by the dewatering wells.

A technical risk analysis of the hydraulic gradient system is the most interesting and challenging of the three systems examined. The basic questions are:

(a) Will contaminated water be capable of flowing between the dewatering wells and thus escape capture by the dewatering system?; and

(b) Given sufficient pumping on the dewatering wells and the recharge wells to prevent leakage through the line of dewatering wells, will the recirculation of clean water through the dewatering wells be acceptable?

The biggest question with respect to a hydraulic gradient concept was whether it would completely intercept the downgradient flow of contaminated water, or whether it would in fact "leak", or pass between, the wells of the dewatering alignment and thus escape treatment. It was our original belief that the originally proposed hydraulic gradient concept would "leak", so an alternative was examined and documented. Technical performance of this alternative is presented in Figures 7 to 11.

However, the recent design of the hydraulic gradient system has been rather loose, and rather than joust with non-specific designs, a small treatise has been prepared covering available options. This is presented in subsequent paragraphs.

Hydraulic Barriers

The simplest model that one can presume for the hydraulic barrier is an infinite, continuous dewatering well (analogous to the French drain, only actively pumping because a physical barrier is absent), and a similar recharge well. Such a configuration possibly could simultaneously trap

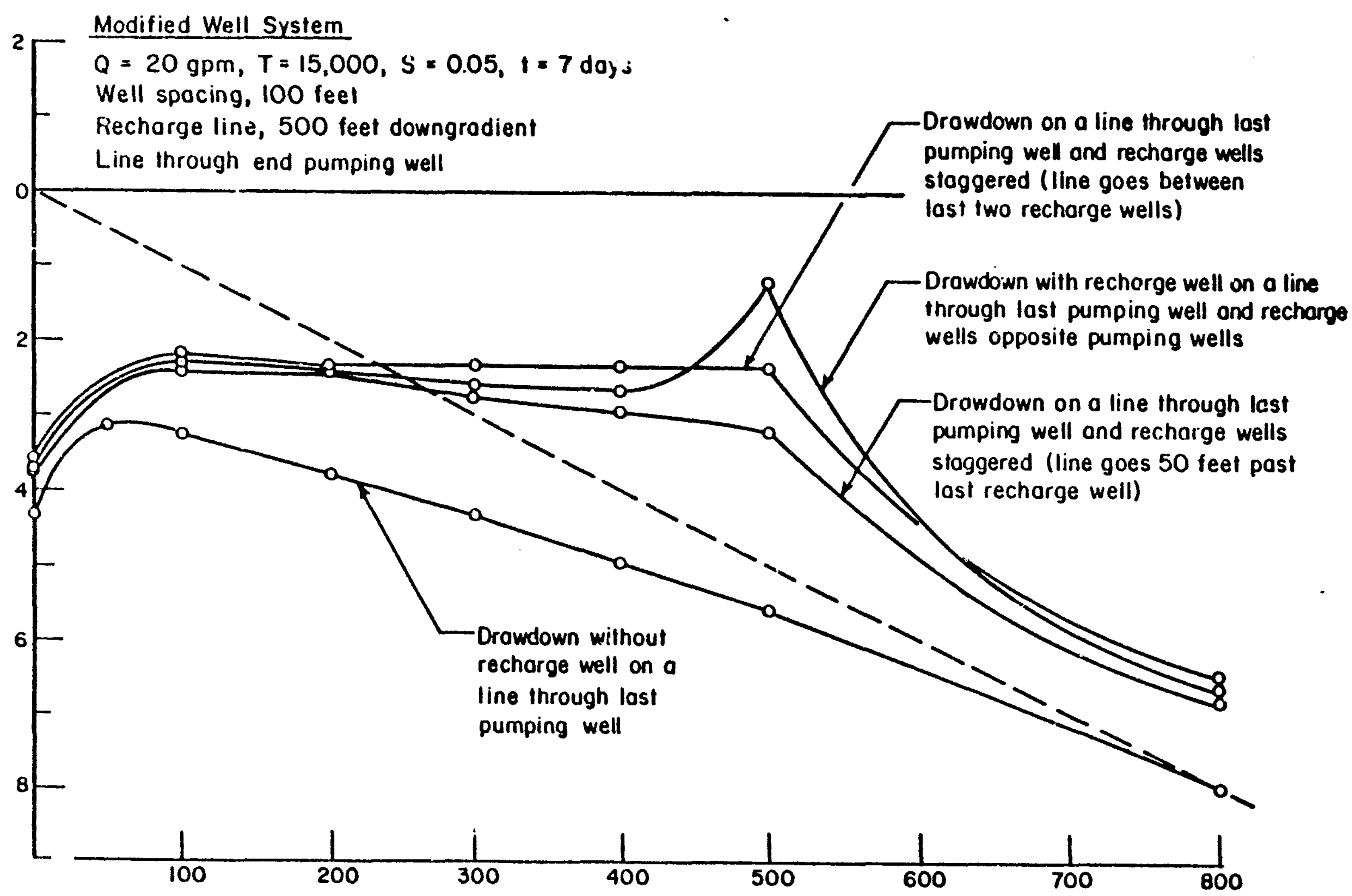


FIGURE 7. PERFORMANCE OF MODIFIED HYDRAULIC GRADIENT CONCEPT

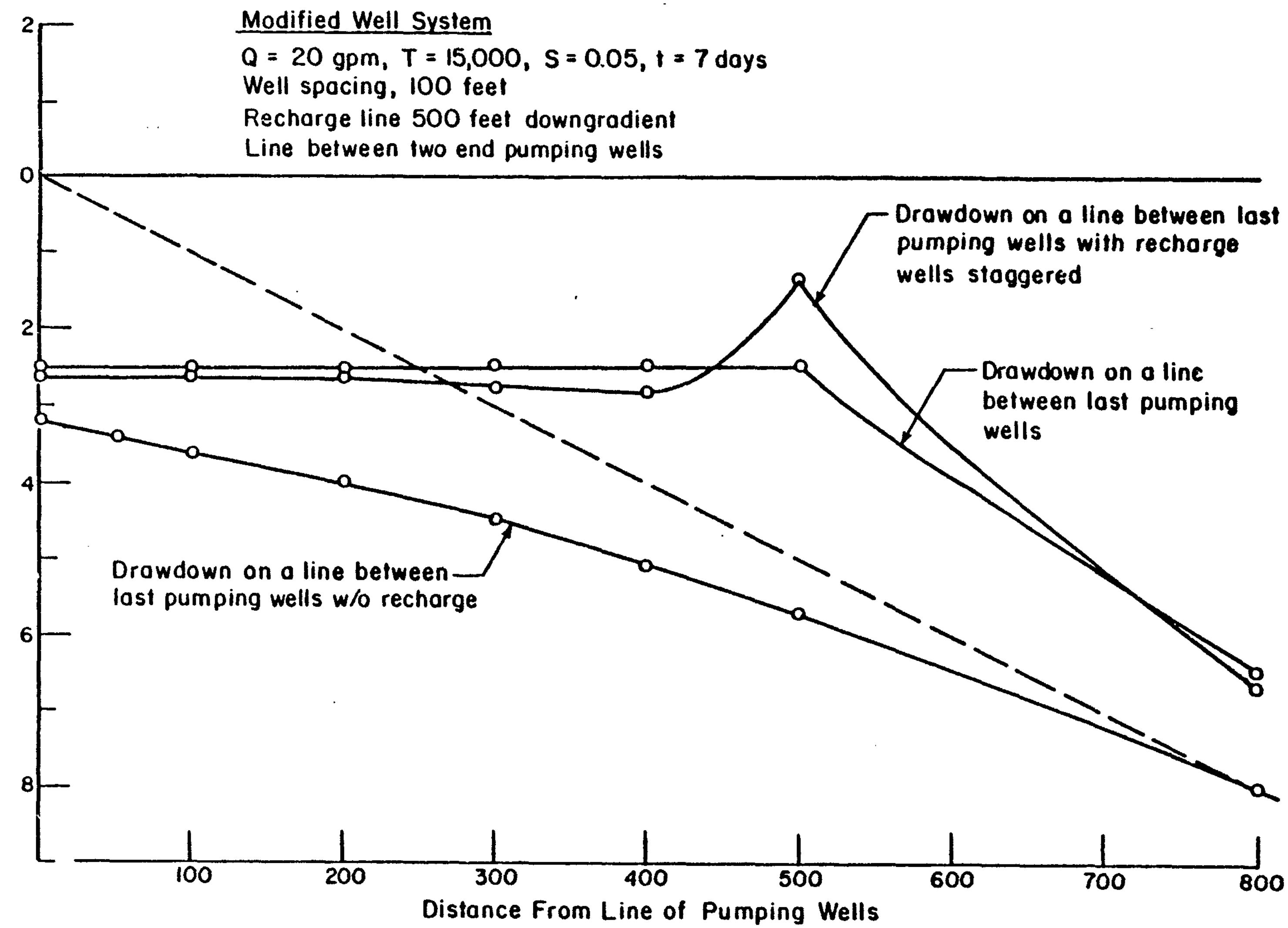


FIGURE 8. PERFORMANCE OF MODIFIED HYDRAULIC GRADIENT CONCEPT

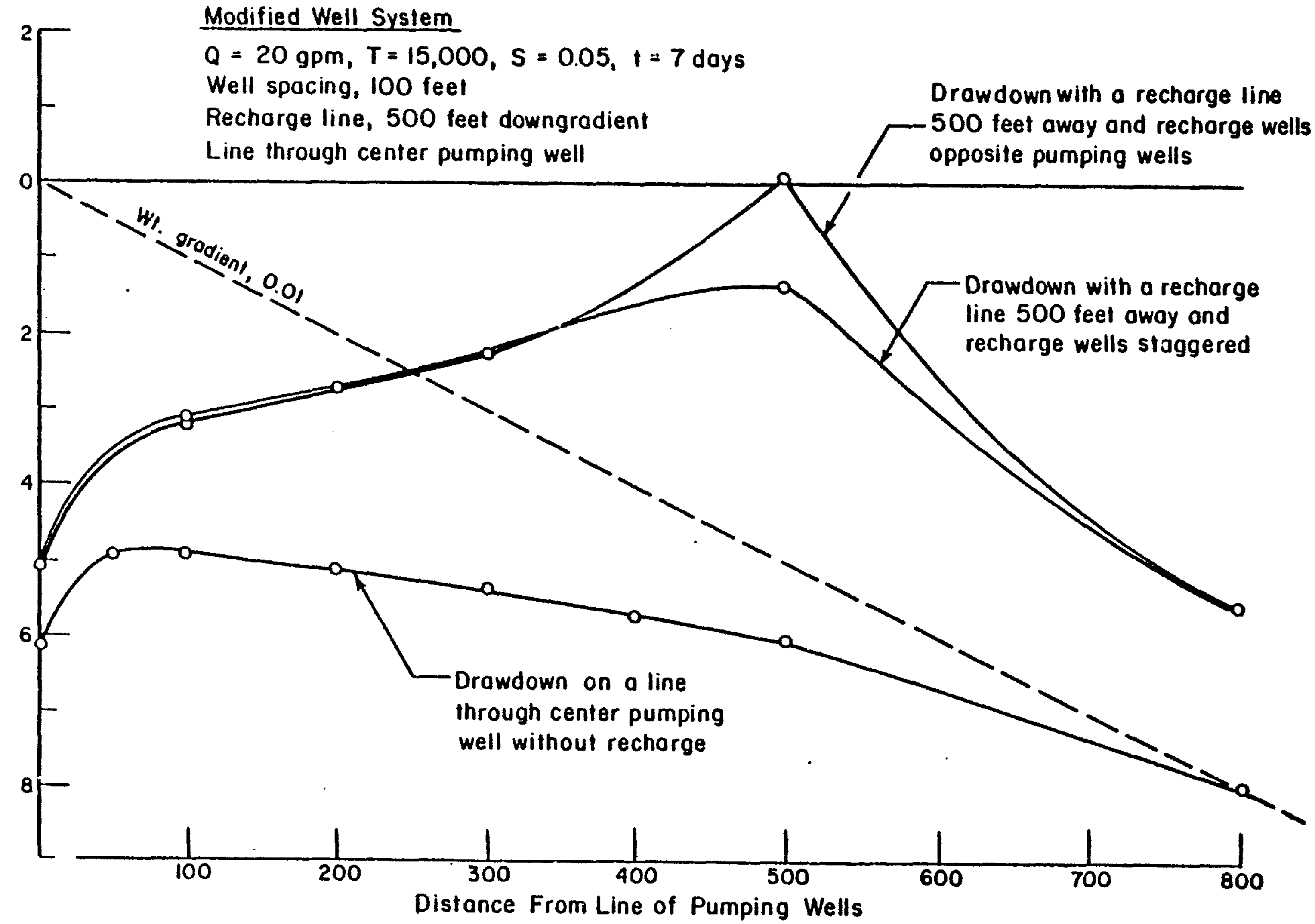


FIGURE 9. PERFORMANCE OF MODIFIED HYDRAULIC GRADIENT CONCEPT

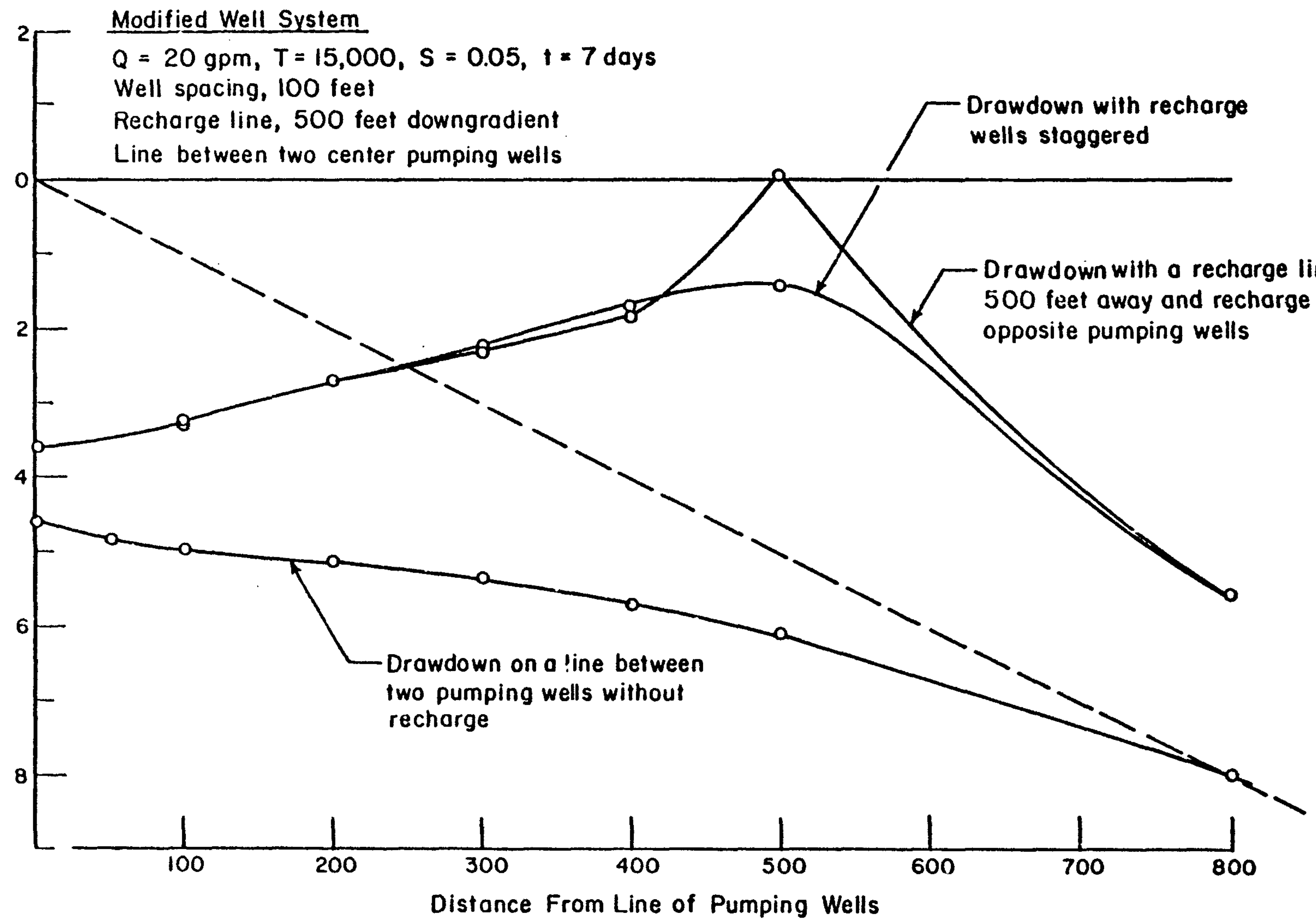


FIGURE 10. PERFORMANCE OF MODIFIED HYDRAULIC GRADIENT CONCEPT

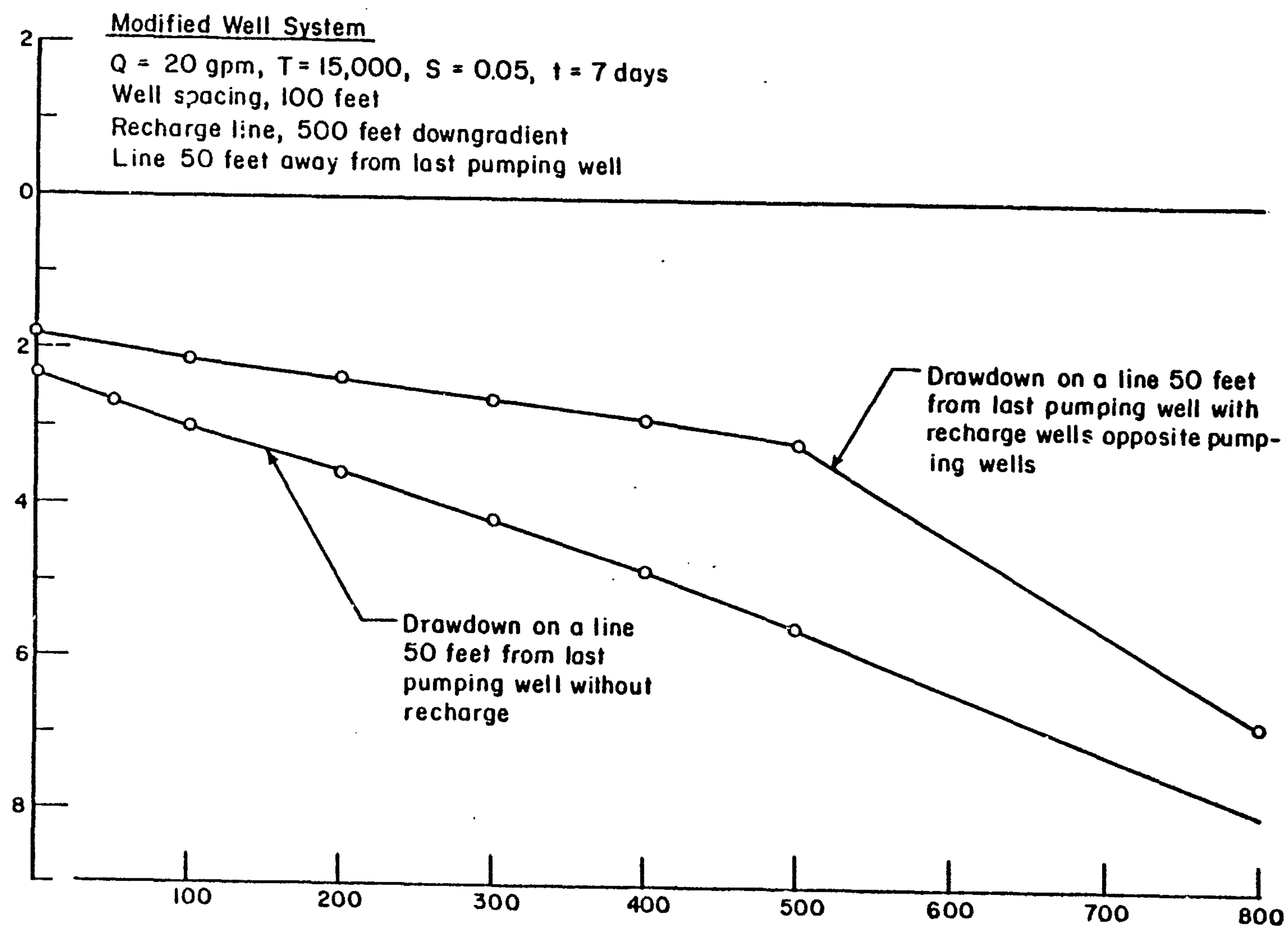


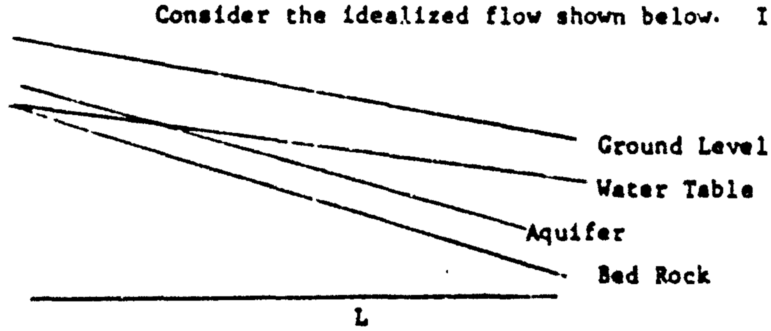
FIGURE 11. PERFORMANCE OF MODIFIED HYDRAULIC GRADIENT CONCEPT

all water coming downgradient while retaining a level water table between the two lines, thus avoiding either downgradient flow of contaminated water or upgradient recycling.

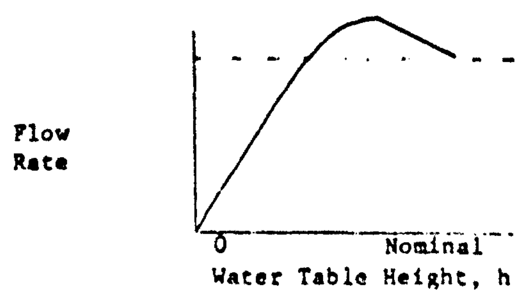
Such a configuration can be represented by a one dimensional analysis. The solution to the governing La Place equation shows that the water table slope induced by a well is linear (as opposed to logarithmic in the 2-dimensional case).

The flow coming downgradient is constant, dependent only upon local rainfall. Therefore, regardless of the pumping rate of the well line in the steady state case, only this flow can be captured from upgradient, and the remainder would have to come from downgradient.

Consider the idealized flow shown below. If one analyzes the



flow out the aquifer as a function of the water table height the results shown below are determined. Since the flow rate is limited, there are only



two values of water table height which will match nominal downgradient flow in a steady state. One value of h allows for no capture: the other is so low that excess recirculation would result.

The gist of this argument is that an infinite well line cannot, in the steady state, simultaneously capture all downgradient flow and stop all flow between the dewatering and recharge lines. An attempt to do so would result in transients which would cause the escape of contaminated water (pumping at the head for a no-recirculation condition would overdraw the aquifer: the enforced pumping pause for recharge would allow the escape).

A more complete analysis was made of the well flow field by modeling the aquifer as 5000 ft wide by 7000 ft long (to the north boundary). The recharge wells were presumed to be placed on the north boundary, while the dewatering well line was placed at varying distances south of the boundary. The dewatering wells, each with discharge rate Q, were placed at equally spaced intervals along the dewatering line, which was 3500-foot long and anchored at the west boundary. The recharge line was similar, except with twice as many wells with each having a recharge rate of Q/2 (in an attempt to model the recharge pits). Reflections of these wells beyond the east, west, and south boundaries of the aquifers were used to represent these impermeable boundaries.

Since there is some disagreement of the value of slope, permeability, and aquifer thickness, nondimensional results were derived from model (derivation in the appendix). By examining the slope of the water table induced by the well system at a point midway between the dewatering wells and midway between dewatering and rewatering alignments (near the end not anchored in the boundary), it was possible to deduce the flow rate Q necessary to stop the leakage of contaminants past the well system. Note that is not correct to examine the saddle drawdown of a dewatering line above and compare it to a similar point on a rewatering line alone: there are interactions between the lines which cause this method to underestimate the Q required.

The results of this analysis are presented in Figure 12. The ordinate of this Figure is the ratio between the well dewatering rate and the flow coming downgradient that the well is to intercept. This is not the flow which can be accomplished: it is the flow which is required to prevent leakage.

If the flow is unobtainable, the system will not work. If it is, then the ordinate represents the volume of clean water circulating with the contaminated water through the treatment system. This Figure also presents the recirculation of the systems, which is approximately the same for well line separations between 250 and 500 feet.

From Figure 12, it is seen that significant excess flow rates are needed regardless of configuration. An analysis of recirculation shows that recirculation will not provide the needed flow: thus the aquifer will be overdrawn. To avoid "mining" the aquifer, then on the average pumping must decrease to facilitate recharge: the decreased pumping will cause leakage according to the formula.

% Leakage = (1 - 1/Total Normalized Flow Required) x 100.

By this analysis, it is seen that for 250' interline spacings and well spacings between 200 ft and 400 ft, one could reasonably expect 11% leakage.* Similarly, for 500' interline spacings and well spacings between 300 and 400 ft, leakage would also average about 11%. It should be noted that a limit to well spacing of about 350-400' exists based upon maximum available drawdown.

Since none of the hydraulic gradient system concepts can operate without leakage in the steady state, one has to reject the hydraulic gradient system concept principle if total control is desired. However, for the concepts in the range of 250-500' interline and 200-400' interwell spacing, the overdraw is small--in the neighborhood of 12%. A passage of several years may be required to significantly impact the aquifer, and thus the hydraulic gradient system concept could operate during this initial transient period (about 1.7 years) without leakage. A higher value of overdraw decreases this transient period of operability (a value of 50% yields about 0.4 years).

Another consideration is that a 12% leakage could be diluted by recharge pits. A discussion of this consideration is included in the Appendix where it is shown that the expected operational mode of the hydraulic gradient system will not allow the 500 ppb limit for DIMP to be met for some intervals of the alignment at a distance 100 feet down-gradient from the recharge pits (for a configuration of 50' interval dewatering wells, on lines 500' apart).

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* Taking into account recirculation. Again, these numbers reflect near-the-edge requirements: requirements of centerwells are smaller.

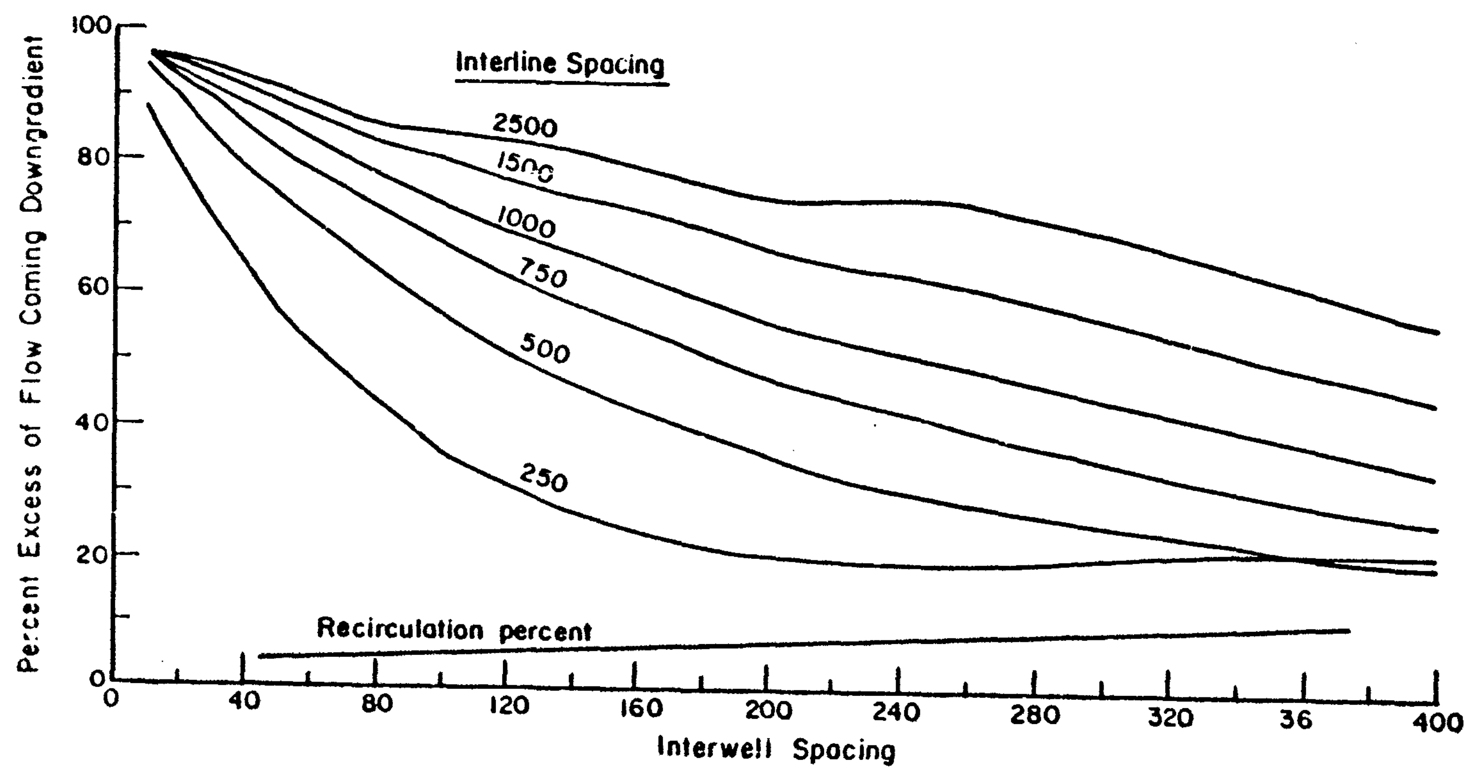

FIGURE 12. PERCENT EXCESS FLOW RATE

The conclusions to this technical risk section are as follows:

- The French drain and bentonite dam systems have high probabilities for success, and very low downside risk
- Hydraulic barriers will leak, but after some initial transient period. Leakage depends upon design, but, even after dilution with recharge water, concentration standards can be violated for some intervals of the alignment.

Selection of Recharge Method

Three systems have been proposed for recharging the processed groundwater, namely:

- Recharge wells
- Surface pits, intercepting the aquifer
- A surface trench (French drain), intercepting the aquifer.

The capital costs estimates for the three systems for a full alignment range between $40,000 to $45,000, and thus there is no essential capital cost difference between recharge systems.

Technically, all systems could work, with the recharge wells being slightly superior because of the ability to return groundwater to the aquifer in exactly the same pattern it was withdrawn. This feature is probably of low value, because the flow patterns downgradient from any of the three recharge systems would tend to replicate the original flow pattern.

The open pit system may be critized on the basis of evaporation and of the potential introduction of microorganisms and oxygen to the aquifer. However, the existing bog already intercepts the aquifer with no harm: hence, any new recharge pit should also be innocuous. Also evaporation from the pits (at the rate of 46"/year typical of the Denver area) would be 1.4 million gallons/year--less than 1% of the total system flow. The open pit and surface trench systems are more conducive to the dilution of any leakage.

Another area to be considered in recharge system selection is operating costs. All three systems are subject to plugging from silt packing and microorganisms. If a well becomes plugged, it must either be replaced or be redeveloped by dewatering. In the meantime, the plugging effects the operation of the remainder of the whole system. If a drain becomes plugged, re-excavation is the cure, with the costs approaching original installation costs. If a pit becomes plugged, the bottom will have to be scraped.

Another point in considering operating costs is that of the operability of the system. To control the recharge wells it will be necessary to couple control devices on each well to the control devices on the dewatering wells (or else the two independent control systems will "fight"* each other). On the other hand, flow to the pits can be regulated by a simple flow splitter. Based on all the above considerations, the open recharge pits are believed to be slightly superior to the other two alternatives.

Recommended System

It was shown that neither the bentonite dam norFrench drain systems, which are technically equivalent, could be built under the capital cost constraint. The bentonite dam is much superior to the French drain, however, based on total installed cost.

A hydraulic gradient system does meet the cost constraint, but was shown to be less than satisfying, technically. There exists in this system considerable risk of "leakage", and also the treatment costs of recirculation (between 5-10%). Although cheaper than the others, the technical risks of the hydraulic gradient system are high enough that it is believed that none of the examined hydraulic gradient systems will achieve the interim standard for contaminant discharge for all points along the alignment. Furthermore, a system which marginally meets the interim standard may have to be scrapped if, in the future, the standard is tightened.

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* Some types of independent controls (well level controllers) will "fight", while others, such as flow splitters, will not.

There is one further--and very important--constraint on deploying a complete north boundary alignment at this time: Namely, treatment capacity is limited to 1000 gph. With this limitation, no system is going to address the entire north boundary by October 1977.

With this in mind, Battelle recommends that a complete north boundary system be installed "in stages". In the first stage, a bentonite dam, dewatering wells, and a recharge pit should be installed and operated for a line sufficient to intercept 10000 gph peak flow (approximately 1500'). After operation of this partial system for a period of time, it should be possible to determine how to satisfactorily complete the system. This segment should be emplaced extending eastward from a point 1400 ft west of 'D' Street, so as to capture the most contaminated flow.

The bentonite dam system is recommended because, for the suggested segment, a hydraulic gradient system will not satisfy downgradient contaminant discharge requirements. This is shown in the Appendix for a complete alignment: a partial alignment (which would be necessitated because of the limited treatment capacity) will leak even more due to edge effects. Groundwater recharge should be accomplished via a recharge pit.

Oversights

Several questions arose during the conduct of this study which need to be addressed prior to construction. These are:

(1) What is to be done with the contaminated silt from recharge pit construction, and excess volumes of contaminated water during well development and pump tests?

(2) Are the possibilities for individual well fluctuations so large that a holding tank needs to be placed in the treatment system?

(3) If recharge wells are used, how can the controls on recharge wells be tied to those on dewatering wells, so as to avoid conflicts between independent controls.

(4) The hydraulic gradient system specifically, and probably the other systems, will tend to pull water from the east. This water normally flows northerly, being divided off the arsenal by a groundwater divide. The consequence of pulling this water to the west would be a decrease of flow to the north, and hence a lowering of water table there. Might it be necessary to recharge the east side via a single recharge well or addition of water to First Creek?

APPENDIX

If there are a number of wells located at (xi,yi) with dewatering rates Qi, then the head at any joint (x,y) in an infinite artesian aquifer is given by the solution to the La Place equation

$$h = \frac{1}{4\pi kb} \sum_{i} Q_i \ln \left\{ (x-xi)^2 + (y-yi)^2 \right\} + C$$

where k = Permeability

b = Aquifier thickness

c = Arbitrary constant

Consider the following well plan

$$\begin{array}{llllllllllll} \text{dewatering} & o & \xleftrightarrow{L} & o & & o & & o & & o & \Big\updownarrow & D \\ \text{recharge} & o & o & o & o & o & o & o & o & o & & \end{array}$$

where the dewatering wells operate at rate Q and the recharge wells at rate Q/2 (twice as many wells).

The goal of this analysis is to determine the flow rate such that the induced water table slope at a point midway between the dewatering wells and the two well lines exactly counterbalances the natural water table slope i. The equation for slope is found by differentiation

$$\frac{\partial h}{\partial x} = i = \frac{1}{2\pi kb} \left\{ Q \sum_{\text{dewater}} \frac{(x - xi)}{(x - xi)^2 + (y-yi)^2} - \frac{Q}{2} \sum_{\text{recharge}} \frac{(x - xi)}{(x - xi)^2 + (y-yi)^2} \right\}$$

$$i = \frac{Q}{2\pi kb} f\ (x,xi,y,yi)$$

where f is terms of the two summations.

Rearrangement yields

$$\frac{Q}{kbiL} = \frac{2\pi}{Lf}$$

In short, this equation defines the needed pumping rate as a function of the available flow, calculable from the geometry of the system.

The discussion above was directed to discrete wells on a finite line, and thus produced results which were not in agreement with the intution of some reviewers. As it turned out, the disagreement resulted from the near-the-end conditions analysis of this report, as contrasted to beliefs of results for near-the-middle wells.

To further amplify the analysis of this report, the limiting case of infinitesimally spaced wells was developed from the preceding equations. The methodology is straightforward, and, in the limit, produces analytically integrable results as follows for semi-infinite dewatering and recharge lines:

$$q/kbi = \frac{2}{1 + \frac{2}{\pi}\,\mathrm{Tan}^{-1} Y/D}$$

where

q = Pumping rate per foot of alignment

Y = Distance in from end of well line

D = Interline spacing.

This function is presented in Figure 13, for a D of 500 feet, and shows that conditions near the end of the line vastly overdraw the aquifer. Overdrawing decreases away from the end, but remains significant to 1000' inward.

Since the aquifer cannot be overdrawn in steady state operation without severe recirculation, it will be necessary to back off on pumping rate, leading to leakage of contaminated water. Using the methodology described in the text, the estimated leakage rate for the continuous well line as a function of distance in from the end is given in Figure 14.

The leakage will underflow the recharge pits (or the continuous well) with little mixing with recharge water. Diffusion mixing will occur as the total water flow moves downgradient.

The model for diffusion processes is the time dependent diffusion equation. A simple representation of the physical processes is to treat the mixing problem as a column, with initial conditions being essentially

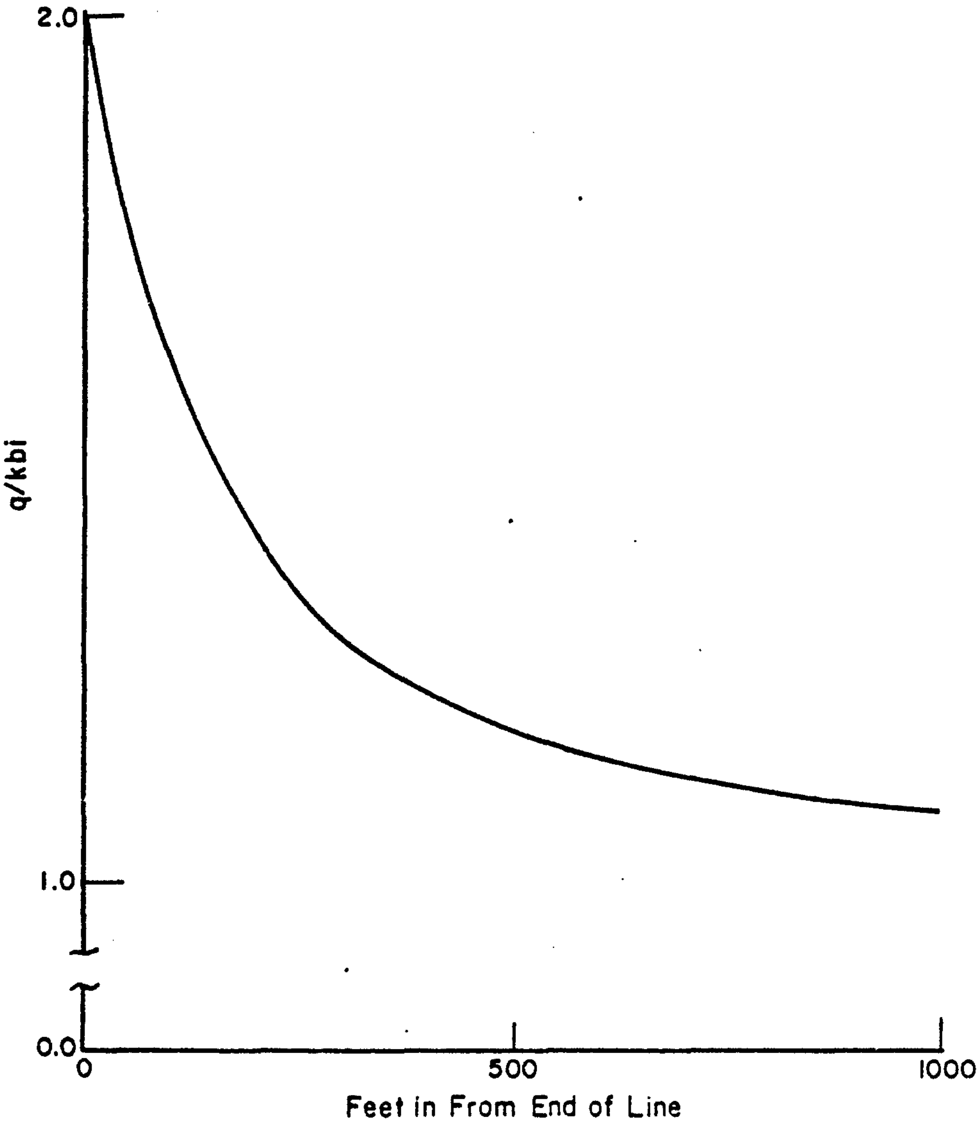

FIGURE 13. Q RATIO REQUIRED FOR SEMI INFINITE LINE DISTANCE IN FROM SENSI INFINITE LINE

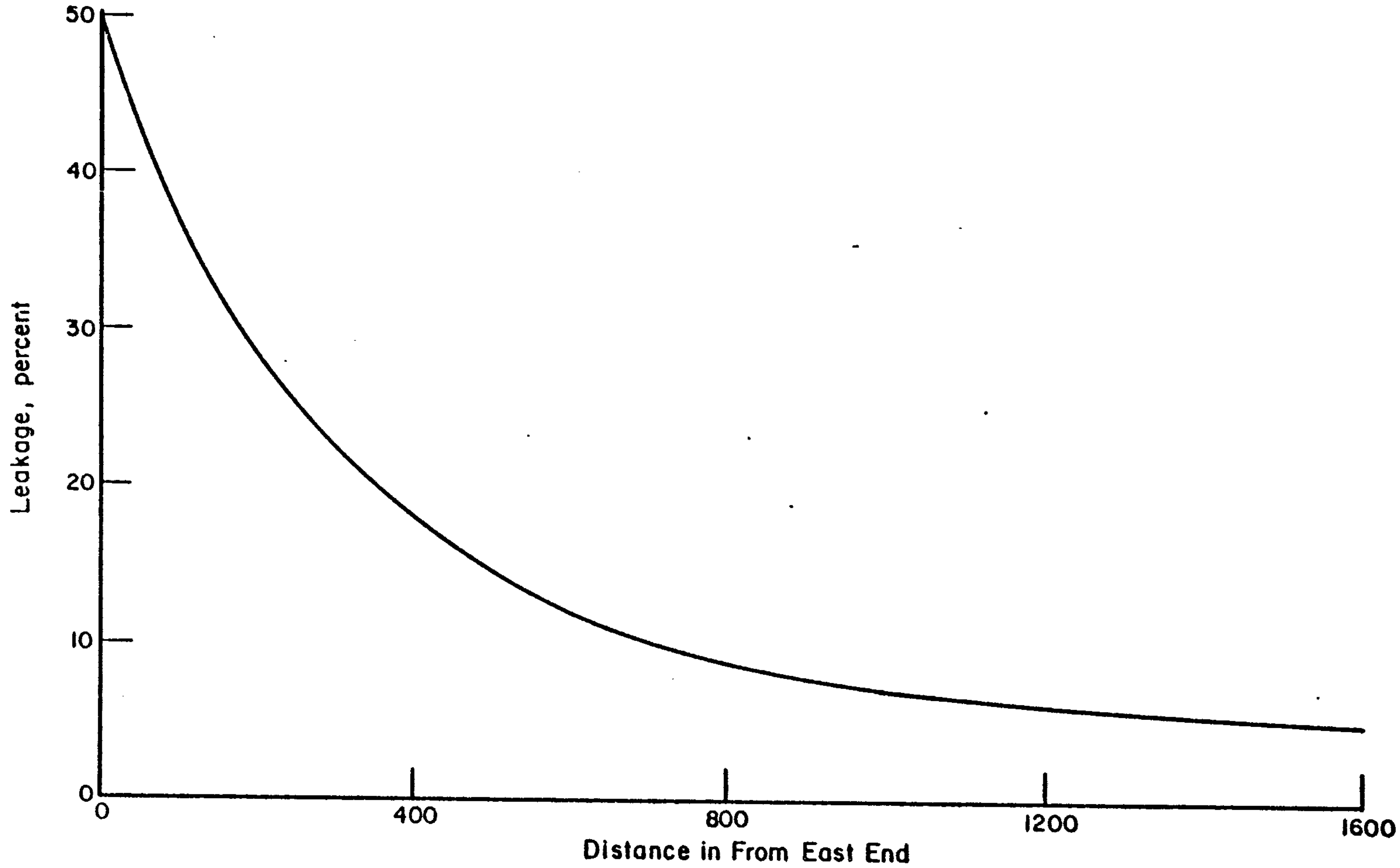

FIGURE 14. SYSTEM LEAK RATE

clean water at one end and essentially contaminated at the other, the ratio of contaminated and clean lengths being given by the fraction of leakage. The solution for contaminant concentration as a function of column distance and time is given by

$$\frac{\chi(x,t)}{\chi_o} = f + \sum_{n=1} e^{-Dn^2\pi^2 t/\ell^2} \left(\frac{1}{n\pi}\right) \cos \frac{n\pi x}{\ell} \sin n\pi f$$

where

f = Leakage fraction

D = Diffusion coefficient (22 ft^2/year*)

ℓ = Column height (10 feet)

χ_o = Contaminant concentration in leakage

t = Time, years

$\chi(x,t)$ = Resultant contaminant concentration profile.

Using a position of 100' downgradient of the recharge system (off arsenal), corresponding to a period of time of 20 days, the dilution of the leaking contamination is illustrated in Figure 15. These results are valid only for recharge pits: contaminant leaking between wells would not be diluted as much as shown here.

By combining the dilution with initial contaminant concentration and treatment efficiency (values of 95% and 100%), applied to the average contaminant loading of 1700 ppb and yielding 85 ppb and 0 ppb, respectively, it was possible to estimate off-arsenal concentrations after installation of a hydraulic gradient system. Three curves are presented in Figure 16: one representing a hydraulic gradient system with 95% treatment, one a hydraulic gradient system with 100% treatment, and one a bentonite dam system with 95% treatment. It is seen that a bentonite dam will be required in places along the alignment to control downgradient concentrations to below 500 ppb.

* The USGS model presumes a value of about 220 for horizontal diffusion. Vertical diffusion in stratified material can be expected to be 1/10th of that value.

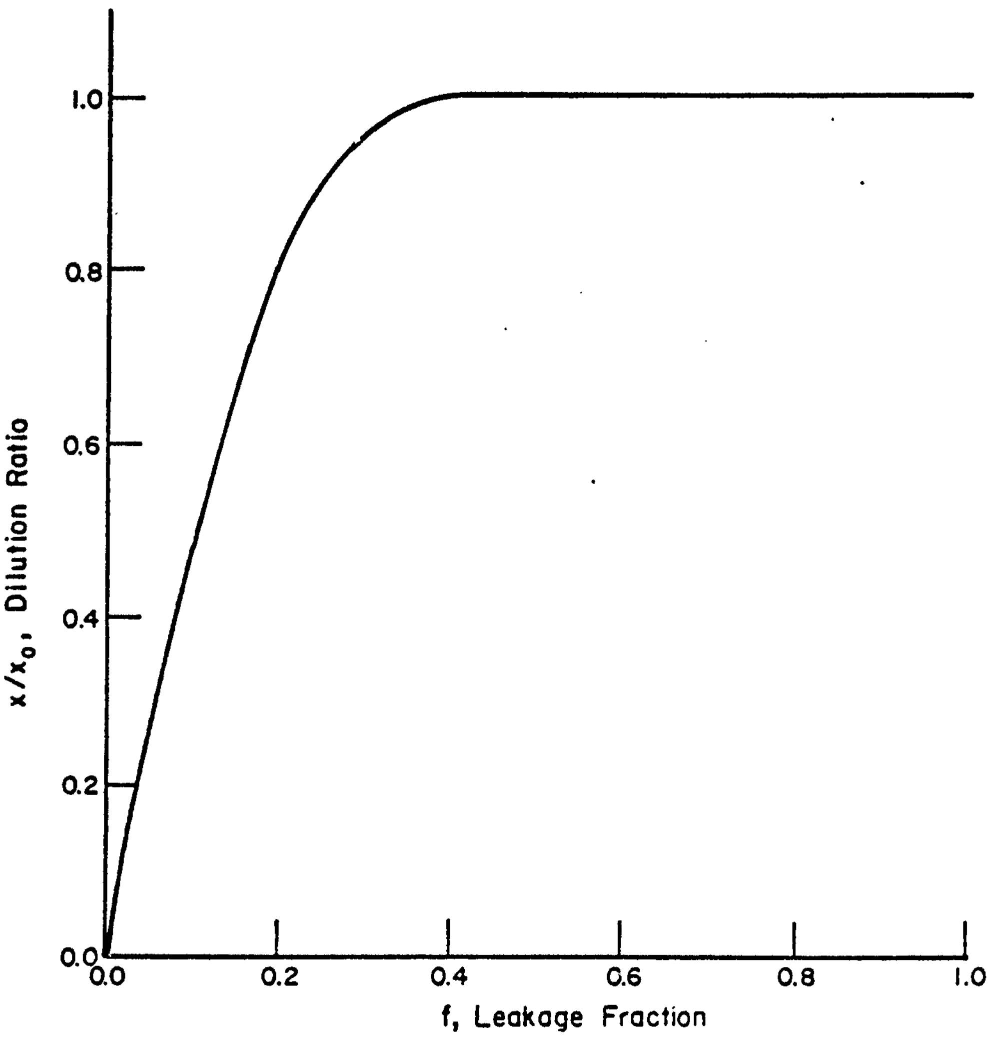

FIGURE 15. DILUTION OF LEAKING CONTAMINANT 100' DOWNGRADIENT

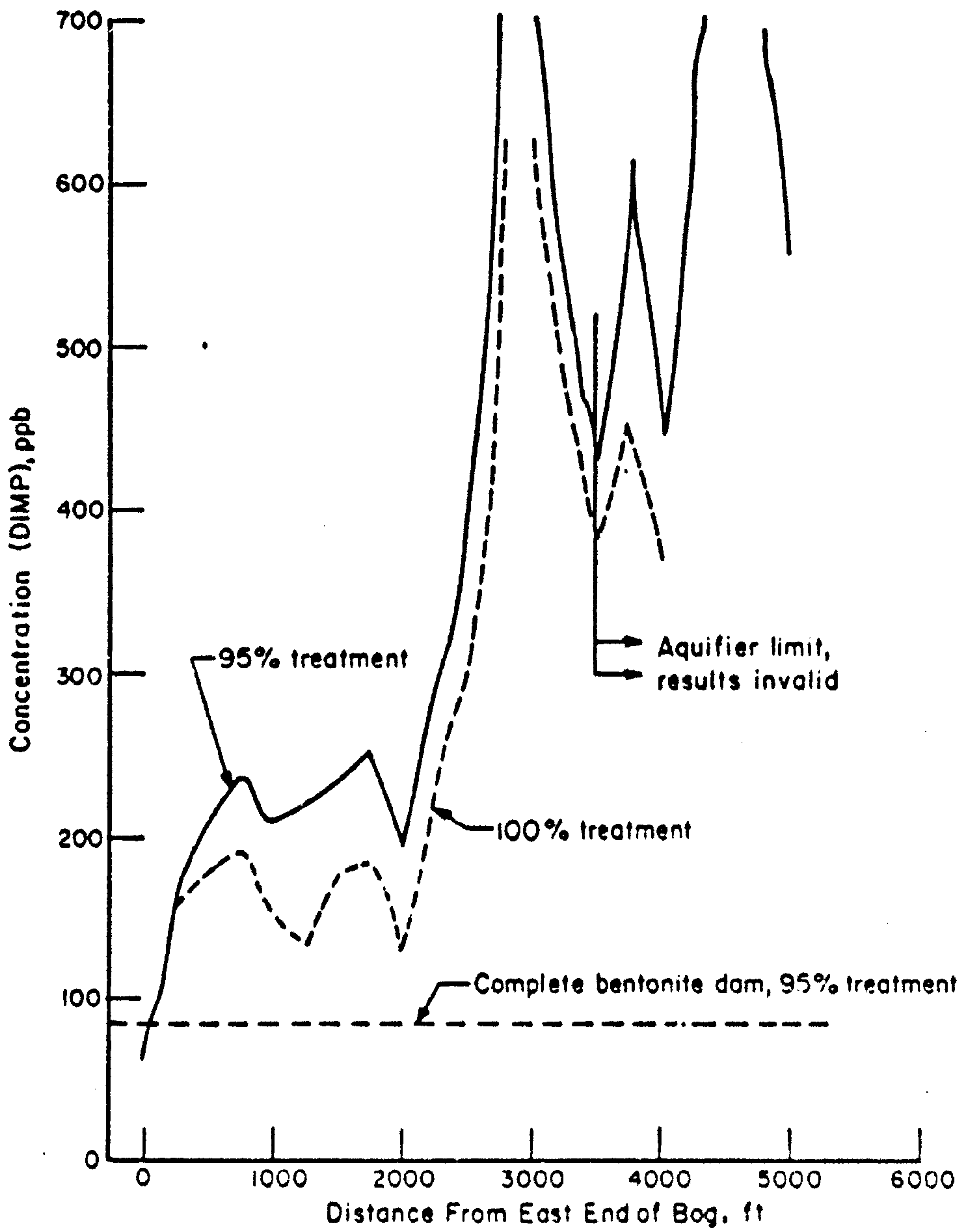

FIGURE 16. DOWNGRADIENT CONCENTRATIONS OF DIMP